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Documenting the Construction of a Plain Concrete Bridge Deck and an Internally Cured Bridge Deck

Carmelo Di Bella

Purdue University, cdibella@purdue.edu

John Schlitter

Purdue University, jschlitt@purdue.edu

Neal Carboneau

Transportation Training Institute, LLC, ncarboneau@gmail.com

Jason Weiss

Purdue University, jason.weiss@oregonstate.edu

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***Documenting the Construction of
a Plain Concrete Bridge Deck
and an Internally Cured Bridge Deck***

**June 2012
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Indiana LTAP Center

**Purdue University School of Civil Engineering
Indiana LTAP Center
3000 Kent Avenue
West Lafayette, Indiana 47906**

**Telephone: 765.494.2164
Toll Free in Indiana: 1.800.428.7639
Facsimile: 765.496.1176**

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16. Abstract: Durable and long lasting concrete is important, especially for bridge decks, which are susceptible to premature cracking and corrosion of the reinforcing steel. To increase the durability of the concrete and its service life, many transportation agencies use high-strength concretes. However, high-strength concretes often have problems with early age cracking due to shrinkage. These cracks can then open a path for chloride ions (found in road salts) to reach the reinforcing steel. The results of this research, which compared bridge installations on two structures in Monroe County, Indiana, confirm that internally cured concrete presents a better alternative to traditional plain concrete for durable bridge decks. With internally cured concrete, a portion of the fine aggregate in the concrete is replaced with the same volume of prewetted lightweight aggregate. As the concrete cures, water from the prewetted aggregate provides the hydration necessary for curing, and also enables curing from the inside. This internal curing process results in a concrete with less initial cracking, less shrinkage, lower thermal stress, lower strain, and greater resistance to chloride ion penetration, with similar or slightly higher strength, relative to plain concrete.			
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Documenting the Construction of a Plain Concrete Bridge Deck and an Internally Cured Bridge Deck

#49 & #61 on Mt. Gilead and Getty's Creek Road



By
Carmelo Di Bella, John Schlitter, Neal Carboneau and Jason Weiss
Department of Civil Engineering
Purdue University,
West Lafayette, Indiana, USA

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1. – Introduction

The desire to provide a durable and long lasting concrete has been the main concern for many transportation agencies. Concrete bridge decks are susceptible to premature cracking and to corrosion of reinforcing steel. Properly constituted, placed and cured concrete bridge deck mixtures are essential to providing long term performance.

In an attempt to increase the durability of the concrete and its service life many transportation agencies have shifted to the use of high strength concretes due to their higher strength and reduced permeability (Bentz, 2000). However, high strength concretes have exacerbated the problem of early age cracking due to self-desiccation that leads the concrete to undergo autogenous shrinkage which, if restrained, may lead to cracking (Weiss, 1999; Jensen et al., 2001). Cracks in concrete represent a preferential path for aggressive species such as chloride ions to reach the reinforcing steel.

In addition to the early age cracking, the durability of the concrete is largely governed by the diffusive transport of ionic species and on the whole by its fluid transport properties. Bridge decks need to provide adequate resistance to chloride ion ingress. In reinforced concrete the ingress of chloride ions reduces the natural passivity of steel reinforcement and promotes the formation of corrosion products which exert tensile forces on the concrete cover and are responsible for causing delamination and spalling (Khan, 2010).

Internal curing by means of prewetted lightweight aggregates represents an effective method to design concrete to be less prone to early age cracking thanks to the additional moisture provided inside the concrete to counteract the effects of self-desiccation (Kovler & Jensen, 2007; Bentz & Weiss, 2010, Schlitter et al., 2010; Weiss, 1999; ACI Committee 213, 2003). In addition to reducing the harmful effect of self-desiccation, internal curing reduces the fluid transport and ionic species ingress thanks to a denser microstructure of the paste and of the interfacial transition zone (ITZ) due to the increased hydration (Castro, 2011). This in turn would result in an increased durability since a denser ITZ can act to depercolate the pores in the matrix.

Recently, there have been efforts to further extend the use of internal curing for field applications; however, the introduction of internal curing appears to be delayed because of unfamiliarity with production, design of the mixtures by practitioners, how to specify these mixtures, and how to quantify the real benefits and costs of using these mixtures in terms of service life and life-cycle cost.

This project was designed to help transfer technology to the field and to provide a side-by-side comparison of the behavior of a conventional (plain) and internally cured concrete in field applications. Two bridge decks were cast in September 2010. One of the bridge decks was cast using normal concrete (plain) and the other bridge deck was cast using internally cured concrete (IC). The internal curing was provided through the use of pre-wetted lightweight aggregates.

This study consists of the evaluation and comparison of two different concrete bridge decks in close proximity to each other which are subjected to the same environment and traffic.

The objective of this report is to document the proportioning, mixing, and casting of two bridge decks and to investigate the impact of internal curing on a variety of concrete properties with regard to volume changes, chloride ions transport and service life.

1.1 – External Curing vs. Internal Curing

External curing is the conventional approach to curing and has been used extensively in the concrete industry. Conventional external curing places water at the surface of the concrete shortly after placement that can be absorbed overtime. In practice, water curing is often difficult to perform and as such, curing membranes or sealers are often used (Figure 1); however, these approaches do not provide additional needed water to the system. Further, when the water cement ratio is low, external curing may be ineffective because of capillary depercolation (Bentz et al., 2005).



Figure 1: External curing by means of pre-wetted burlap on the internally cured bridge.

Internal curing refers to the use of prewetted fine lightweight aggregate (LWA) or any other water inclusions to provide moisture to the hydrating cement throughout the cross section of the concrete after setting. Prewetted lightweight aggregate is used in place of a fraction of the conventional sand used in a mixture. Its main feature is the high porosity that will allow storage of water and acting as reservoirs able to release water whenever the system needs it. While the water remains primarily in the prewetted LWA during mixing and transportation, the water can be drawn out of the LWA due to pressure that is developed in the pore fluid (e.g., “water” in the concrete) after setting, or prior to setting in the case of evaporation. Figure 2 shows a schematic representation of the difference between external and internal curing.

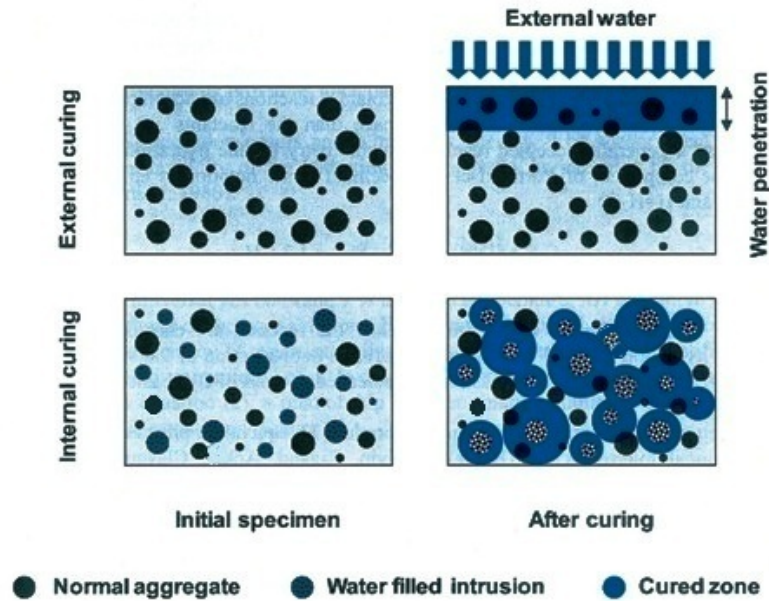


Figure 2: Illustration of the difference between external and internal curing (Castro et al., 2010).

1.2 – Internal Curing

Initially, internal curing was adopted as a strategy to mitigate self-desiccation especially in mixtures with low water to cement ratio. During hydration, the cement paste undergoes a volume reduction which is defined as chemical shrinkage. Before setting, chemical shrinkage results in an actual reduction of the bulk volume. After setting, however, the skeleton formed is strong enough to resist a large portion of the volume change caused by chemical shrinkage (Sant et al., 2006). As a result, vapor filled pockets start to form or in other words some partially empty pores between cement grains are formed (Couch et al., 2006). This is known as self-desiccation. The size of the partially filled pores is important since it will influence the pressure that develops in the system and in turn the magnitude of the autogenous shrinkage (Lura et al., 2009).

Water filled inclusions, such as prewetted lightweight aggregate, are able to counteract the reduction in internal relative humidity providing curing water to the system minimizing the effects of autogenous shrinkage cracking compensating for the volume of chemical shrinkage. The water stored in the lightweight aggregates is typically stored in pores that are larger than those in a hydrating cement paste. As a result the water will move from the LWA to the surrounding paste keeping the small pores saturated (Lura, 2007; Henkensiefken, 2008).

In addition to positively impacting autogenous shrinkage, internal curing by means of pre-wetted lightweight aggregates has also shown to reduce drying shrinkage cracking (Henkensiefken, et al., 2009), the likelihood of plastic shrinkage cracking (Henkensiefken et al., 2010) and the likelihood of thermal cracking (Schlitter et al., 2010).

More recently, it has been observed that internal curing is able to reduce the fluid transport and ion diffusion in concrete. Chlorides ingress is of particular interest for bridge decks in chloride exposed environments. If the chloride ions reach the reinforcing steel they can reduce the

passivity of the reinforcing steel and promote the formation of corrosion products. Corrosion is a major concern since it can shorten the service life of reinforced concrete structures.

The fluid transport in internally cured concretes is reduced in three ways. First, internal curing supplies additional water that promotes increased hydration thereby reducing the porosity of the concrete (Castro, 2011). Second, internal curing reduces the influence of the interfacial transition zone causing a denser ITZ at the LWA when compared with the interfacial transition zone around sand (Zhang et al., 1990; Wislow et al., 1994). A more dense ITZ region would likely provide higher resistance to ionic and fluid transport because it can act to depercolate the paste (Bentz et al., 2000). Third, internal curing reduces unwanted cracking thereby reducing other paths for fluid to reach the reinforcing steel (Cusson et al., 2008; Schlitter, 2010; Raoufi, 2011).

In order to assess the long term performance and to predict the service life of concrete structures chloride diffusion coefficients become a necessary component.

2. – Mix Proportion for Internally Cured Concrete

Usually three critical factors need to be carefully evaluated when using internally cured concretes: 1) the amount of internal curing water provided by water held within the LWA, 2) the ability for internal curing water to leave the LWA's pores and replenish water needed in the system (desorption), and 3) the distribution of the internal curing reservoirs throughout the concrete.

One challenge when proportioning an internally cured mixture with lightweight aggregate is to determine the amount required to provide an adequate supply of internal curing water. Based on the concept of filling the volume of pores created by chemical shrinkage with water from lightweight aggregate Bentz and Snyder developed a straightforward approach simply by equating the water demand of the hydrating mixture to the supply that is readily available from the internal reservoirs (Bentz et al., 1999). The amount of lightweight aggregate inside the mix design can be addressed by equation 1:

$$M_{LWA} = \frac{C_f \times CS \times \alpha_{max}}{S \times \phi_{LWA}} \quad \text{eq. 1}$$

Equation 1 estimates the internal curing water or in other words the mass of dry fine LWA needed per unit volume of concrete (M_{LWA}) required to maintain complete saturation within the hydrating cement paste, by exactly compensating for the chemical shrinkage (CS) of the hydrating cement paste in the concrete mixture at the maximum expected degree of hydration (α_{max}); S and ϕ_{LWA} are the degree of saturation and the sorption capacity of the lightweight aggregate, respectively.

It should be noted that this approach may overestimate the amount of water required for internal curing because it considers the volume change that occurs before set. Other approaches were suggested accounting for the water that is lost from the smallest pores, accounting for the water lost during evaporation or for higher water to cement ratio mixtures (Castro et al., 2011; Henkensiefken et al., 2009). Others have accounted for differences in initial moisture conditions

(Golias, 2010). However those are beyond the scope of what should be used for initial implementation.

3. – Field Location and Bridge Details

A county roadway project involving the replacement of two bridges was chosen for this project about five miles northeast of Bloomington, Indiana. The map in Figure 3 shows the relative location of both bridges 49 & 61 on Mt. Gilead and Gettys Creek road that carry two county roads over a creek.

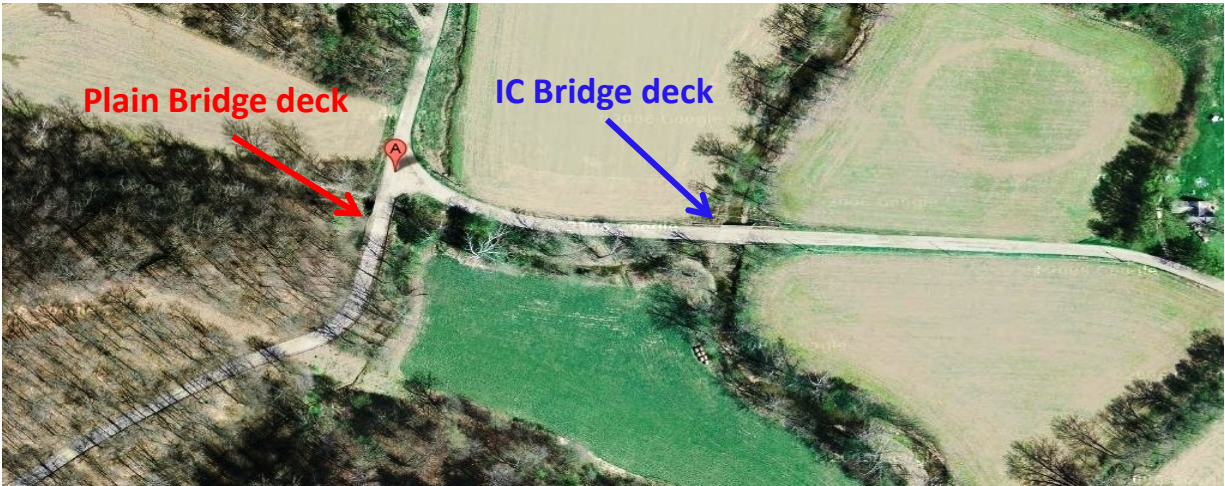


Figure 3: Location of the two bridges; on the left the plain bridge deck and on the right the internally cured one.

Both bridges are similar in structural design and utilize pre-stressed concrete box beams topped with a composite reinforced concrete deck resulting in a 27'-8" wide road surface. In both cases the concrete deck was 8" thick at the roadway centerline and 4 ½" thick at the edge gutters (Figure 4). Epoxy coated deformed reinforcement bars were located at mid height of the decks with chairs (Figure 5). Photos of the typical box beam arrangement prior to casting the decks are shown in Figure 6 and 7. The individual box beams were locked together to form a continuous unit with grouted keyways and 1" diameter transverse through rods. The west bridge carrying N Mount Gilead Rd consists of CB 21"x48" box beams spanning 50'-0" and received a deck consisting of standard bridge deck concrete. The east bridge carrying Gettys Creek Road consists of CB 17"x48" box beams spanning 39'-0 ½" and received a deck consisting of an internally cured mixture utilizing prewetted lightweight sand aggregate (LWA).

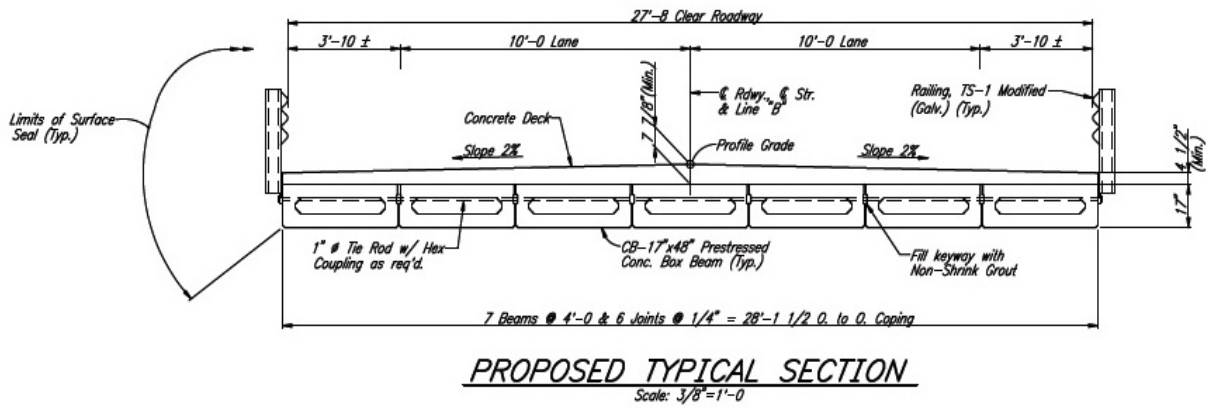


Figure 4: Typical section of the bridge deck.



Figure 5: Box beams before casting the internally cured mixture with formwork and epoxy coated reinforcing steel.



Figure 6: Box beams before casting the internally cured mixture.



Figure 7: Box beams before casting the plain mixture.

The unique feature of the project allows the monitoring of the two bridge decks' behavior under similar conditions, such as climate conditions and traffic.

The concrete mixtures for the decks of both bridges was produced at a local ready mix concrete plant and consisted of locally produced aggregates. The contractor selected their own mixture proportions for the plain deck. These proportions were to be consistent with contract documents as well as INDOT class C mixture requirements. Purdue assisted the contractor in using equation 1 to adjust their plain mixture to include internal curing. These proportions are described later in this report and the specifications are provided in the appendix.

The goal of this study was to document the proportioning, mixing, and casting of an internally cured mixture in the field and to observe the in-situ performance of the bridge decks over time. Further, samples of both mixtures were acquired during casting in order to characterize and compare their performance in the laboratory at Purdue University.

The bridge decks were cast on the subsequent mornings of September 23rd and 24th 2010. The box girders were wetted prior to casting per standard procedure. The plain concrete deck was placed by means of a pump truck due to the slightly longer span (Figure 8 and 9). The internally cured concrete deck was placed using a bucket as shown in Figure 10.



Figure 8: Plain concrete deck was placed by means of a pump truck.

After placement, the fresh concrete was consolidated with mechanical spud vibrators and a gasoline powered vibrating screed was used. The surface was then bull floated, brushed, and tined. Curing on both bridges consisted of wetted burlap as shown in Figure 1 that was maintained for 7 days periodically wetted by the construction personnel.



Figure 9: Plain concrete deck was placed by means of a pump truck.

Initially it was thought that pumping the concrete could result in water preferentially squeezed into the pores of the lightweight aggregate. For this reason the material was placed using the bucket. Since the time of this project, discussions with the NYSDOT have reported that they have experienced no difficulties in pumping internally cured mixtures. As such, pumping would be permitted for future internal curing applications.



Figure 10: Internally cured bridge deck placed by means of a bucket.

4. – Constituent Materials

The concrete mixture used for the plain bridge deck is commonly used by the local ready mixed concrete provider. The second mixture (internally cured) is similar; however a portion of the volume of the fine aggregate has been replaced with an equivalent volume of prewetted lightweight aggregate. Table 1 shows the mixture proportions:

Table 1: Mixture Proportions for Plain and Internally Cured Bridge Decks

	Cement Content (kg/m ³)	W/C	Fine Aggregate (kg/m ³)	Fine LWA (kg/m ³)	Coarse Aggregate (kg/m ³)	Mixture Water (kg/m ³)	Water in LWA (kg/m ³)	WRA (%) ^A	AE (%) ^A
Plain Concrete	390	0.39	726	-	1046	152	-	0.22	0.08
Internally Cured Concrete	390	0.39	313	270	1046	152	25	0.22	0.08

^A Percentage referred to the cement weight.

These proportions are based on the saturated surface dry conditions of the materials with the exception of the LWA which is given in oven dry condition. An additional column is used to represent the water to maintain the 24 hour prewetted surface dry conditions.

The lightweight aggregate (LWA) used in this study was a manufactured expanded shale from Brooklyn, IN. The material is known commercially as Haydite AX fine aggregate in Indiana. The absorption of lightweight aggregate is based on a test procedure from the Department of Transportation for the State of New York (NYSDOT, 2008). After 24 hours, the water is decanted and the surface of the aggregate is patted dry. In this method, the aggregates are spread out, and a paper towel is laid across the surface of the aggregates. Once it appears that the paper towel is no longer picking up moisture (as determined by visual inspection for a change in color from the paper towel), it is assumed that a surface dry condition has been reached, and the aggregate moisture can be determined. The 24 hours absorption of the LWA was 10.4 %. The specific gravity of the lightweight aggregate was 1.56.

The absorption of the normal weight aggregates was evaluated at 24 hours at SSD condition. The absorption was measured in the laboratory according to ASTM C 127 and ASTM C128. The absorption of the coarse and fine aggregate was 1.1% and 1.6% respectively. The specific gravity of the coarse and fine aggregate was 2.672 and 2.647 respectively.

According to the LWA's absorption and the chemical shrinkage assumed to be 0.064 (lb/lb), 57% of fine aggregate was replaced with LWA, for a total replacement in the mix corresponding to 17% by volume.

The cement used for the plain and internally cured mixtures is an ordinary portland cement (OPC) conforming to ASTM C150 type I/II specifications. The characteristics of the cement are given in tables 2 and 3:

Table 2: Characteristics of Ordinary Portland Cement Used for the Bridge Decks

	SiO ₂	Al ₂ O ₃	Fe ₂ O ₃	CaO	MgO	SO ₃	Na ₂ O Equiv.	Blaine (m ² /kg)
OPC Type I/II	20.4%	4.8%	3.2%	63.2%	2.1%	3.4%	0.67%	398

Table 3: Bogue Calculations of Ordinary Portland Cement Used for the Bridge Decks

	C ₃ S	C ₂ S	C ₃ A	C ₄ AF
OPC Type I/II	56%	16%	7%	10%



Figure 11: Lightweight, coarse and fine aggregates – Prairie Material in Bloomington, IN.

5. – Mixing, Curing, and Sample Preparation

In order to perform additional tests a series of laboratory samples were cast using the same constituent materials (Figure 11).

Prior to mixing the lightweight, coarse, and fine aggregates were dried in a oven at 105 ± 5 °C for 24 hours. After cooling, the lightweight aggregate was soaked in the mixture water for 24 hours being careful that the water remained above the top surface of the lightweight aggregate. In order to avoid loss of evaporation the lightweight aggregate was covered with plastic sheets. After 24 hours the excess water was decanted and used in mixing.

The coarse and fine aggregate were placed in the pan mixer. Water was then added to the mixture (along with lightweight aggregate in the case of the internally cured mixture). Cement and admixtures were then added. After all the materials were placed in the mixer they were mixed in accordance with ASTM C192-07.

Cylindrical and prismatic samples were tested. The cylinders had a diameter of 102 mm (4 in) and length of 204 mm (8 in). The prisms were 70 mm (3 in) square in cross-section and

approximately 280 mm (11 in) long. External vibration was used to consolidate the specimens and immediately after casting, they were covered. These specimens were demolded after one day. The cylindrical samples were sealed in plastic bags following demolding, until the associated test ages were reached (with the exception of the surface resistivity samples); the prismatic samples were directly stored in a chamber at 23 °C and 50% relative humidity. At the time of testing the samples were prepared using the procedures described in the following section.

6. – Experimental Program

In order to quantify and compare the performance of the conventional and internally cured bridge decks, several tests were performed with up to a one year aging period. Field samples prepared at the time the decks were cast and laboratory prepared samples were tested at different ages.

First, the compressive strength results of cylindrical specimens cast in the field are presented. Second, volume change properties (autogenous and drying shrinkage) from laboratory prepared mixtures and field cast samples are presented. Third, chloride transport performances are evaluated using a series of experimental techniques including: 1) rapid chloride penetration (RCPT), 2) surface resistivity, 3) rapid migration test, 4) migration cell, 5) chloride ponding and profiling.

6.1 – Compressive Strength

The compressive strength was monitored during the first 3 months in accordance with ASTM C39-01. The samples were directly prepared in the field at the time the decks were cast. After casting, the samples were stored in sealed plastic bags.

The test method consisted of applying a compressive axial load to the molded cylinder, 102 mm (4 in) in diameter and 203 mm (8 in) in depth until failure occurred (Figure 12).

The compressive strength of the cylinders was measured at 3, 7, 14, 28 and 90 days and calculated as an average of three cylinders for each age.

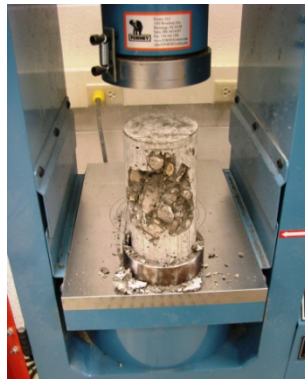


Figure 12: Compression test.

6.2 – Volume change

6.2.1 – Unrestrained Linear Autogenous Shrinkage – Large Tube

The autogenous shrinkage of concrete was measured using a modified version of the ASTM C1698-09 standard. A special corrugated mold was employed to measure the autogenous strain of concrete (Tian et al., 2008). The tube is 300 mm long and has an inner diameter and outer diameter of approximately 60 mm and 80 mm respectively.

Fresh concrete was cast in the mold approximately 15 minutes after the water contact. The concrete was compacted at each lift with rods and vibrated on a vibrating table. The specimens were placed in a dilatometer instrumented with two LVDTs (Figure 13). Measurements were captured every 5 minutes for the first 5 days.

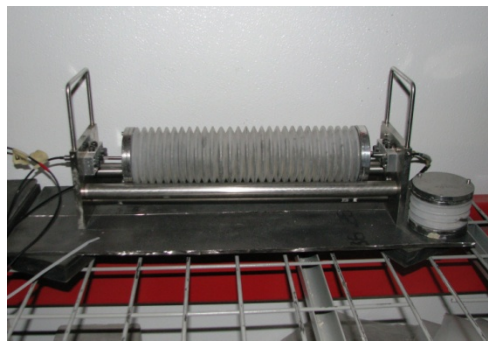


Figure 13: Corrugated plastic tube.

6.2.2 – Restrained Shrinkage—Dual Ring Test

The restrained shrinkage behavior and the reserve cracking strength were quantified with a modified version of the standard restrained ring test (ASTM C1581)—the dual restraining ring test (Schlitter et al., 2010).

The dual ring test is performed by casting an annulus of mortar between two concentric restraining rings that provide restraint to both expansion and shrinkage (Figure 14). The surface of the restraining rings is coated with a layer of acetate sheet in order to minimize friction on the sample. A temperature control system may be utilized with the dual ring test to impose thermal stress on the specimen by lowering the temperature at selected times. The dual ring specimens tested in this report were maintained at 23 °C during the first four days and then cooled to -5 °C. The stress history of the specimen is calculated from the measured strain of the restraining rings that is recorded every five minutes with four equally spaced strain gages on the inner and outer rings. As a result, the calculated stress in the specimen may then be correlated to a given reduction in temperature (more details can be found in Schlitter et al., 2010).



Figure 14: Mortar between the two rings in the insulation chamber.

6.2.3 – Drying and Autogenous Shrinkage

Drying shrinkage was measured in accordance to ASTM C157-08. This test procedure determines the length changes produced by causes other than externally applied forces and temperature changes in concrete.

The test specimens were prepared in the field at the time the decks were cast and consist of prisms, 70 mm (3 in) square in cross-section and approximately 280 mm (11 in) long. These specimens were stored in a chamber at 23 °C and 50% relative humidity. The length was monitored at different ages: 3, 7, 14, 28, and 150 days.



Figure 15: Concrete prism in the length comparator.

6.3 – Transport Properties

6.3.1 – Rapid Chloride Penetration Test (RCPT)

The rapid chloride penetration test (RCPT) was performed in accordance with ASTM C 1202-12. A cylindrical specimen was prepared that was 102 mm (4 in) diameter and 51 ± 2 mm (2 in) long. Prior to testing the sample was vacuum saturated as described in ASTM C 1202-12. During the test, one surface of the sample was exposed to a sodium chloride solution (3% NaCl) and the

other surface was exposed to a sodium hydroxide solution (0.3 M NaOH). A 60 V externally potential was applied and the resulting current at 15 minute intervals was recorded for a six hour period. Figure 16 shows the experimental set up with four samples being run simultaneously.



Figure 16: Rapid chloride Penetration Test cells.

6.3.2 – Surface Resistivity

An easier, faster, non-destructive test method which is an alternative to the RCPT is the electrical surface resistivity test. The electrical surface resistivity of the water-saturated concrete sample was measured using the four-point Wenner probe surface (Figure 17). Measurements of electrical resistivity were carried out as described in AASHTO TP 95-11 using cylindrical samples that were prepared in the field during the construction of the bridge decks and were 102 mm (4 in) in diameter and 204 mm (8 in) long. The samples were demolded and kept saturated in lime water at a temperature of 23 ± 1 °C during the entire testing period. It should be noted that by storing the samples under water it is believed that the samples will absorb water during the test which may increase the degree of hydration of the specimen (Di Bella et al., 2012).

In the surface resistivity test, an alternating current (AC) is applied at outer pins which generate a current flow in the concrete. The potential difference is measured between the two inner pins. The resistivity is then calculated according to the specimen geometry. Assuming that the concrete has homogeneous semi-infinite geometry and the probe depth is far less than the probe spacing, the concrete cylinder resistivity becomes:

$$\rho = (2 \cdot \pi \cdot a) \cdot \frac{v}{i} \quad \text{eq. 2}$$

where a is the electrode spacing, v is the voltage and i is the current.

The surface resistivity of a 4 x 8 cylinder can be related to the bulk resistivity by using a geometry correction (K) of approximately 1.9 for this probe geometry and specimen size (Morris et al. 1996 and Spragg et al. 2011). Equation 3 illustrates this correction:

$$\rho_{bulk} = \rho_{surface} / K \quad \text{eq. 3}$$

Relationships have been developed between RCPT and surface resistivity test results (assuming saturation) since both tests are based on measures of the electrical resistance of the concrete

(Kessler et al., 2008; Spragg et al., 2011). Corrections for saturation are provided in Weiss et al. (2012).



Figure 17: resistivity measurement with a Wenner array.

6.3.3 Rapid Chloride Migration Test

The Nord Test method (NT Build 492) was used in order to determine the non-steady state chloride migration coefficients.

The NT Build 492 is a typical non-steady state migration test to accelerate the chloride transport by means of the application of a potential across a 50 mm thick specimen for a specified period of time.

The test specimens, 51 ± 2 mm (2 in) thick cylinders and 102 mm (4 in) diameter, are vacuum saturated in accordance with NT Build 492. The sample is then placed in a rubber sleeve that works as a reservoir of 0.3 M NaOH on the upper surface of the sample. The bottom of the sample is placed in 10% NaCl. An initial potential of 30 V is applied to the specimen. This potential is adjusted according to the current response as outlined in the standard and the applied potential is maintained for a 24 hour period (Figure 18).

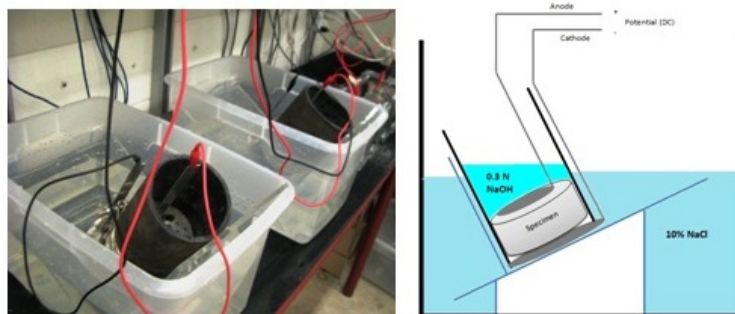


Figure 18: Rapid Chloride Migration (RCM) experimental set up.

At the end of the test period the sample is rinsed with distilled water and the surface is wiped with a cloth. The sample is axially split into two pieces. A 0.1 M silver nitrate solution is sprayed on the sample. Where sufficient chloride is present, the silver nitrate causes white silver chloride

to precipitate (see Figure 19) and the penetration depth of chloride is measured at 10 depths across the concrete piece's section.



Figure 19: Axially split sample after the rapid migration test and sprayed with Silver Nitrate (AgNO_3). The white portion represents the chloride front penetration.

The chloride non steady state migration coefficients D_{nssm} are calculated using equation 4, derived by the non steady state equation for diffusion and migration. The migration process is assumed as dominant and the chloride binding capacity during the test is assumed to be constant.

$$D_{nssm} = \frac{0.0239(273+T)L}{(U-2)t} \left(x_d - 0.0238 \sqrt{\frac{(273+T)Lx_d}{U-2}} \right) \quad \text{eq. 4}$$

where D_{nssm} is the non-steady state migration coefficient ($\times 10^{-12} \text{ m}^2/\text{s}$), U is the absolute value of the applied voltage (V), T is the average value of the initial and final temperatures in the solution (C), L is the thickness of the specimen (mm), x_d is the average value of the chloride penetration depth (mm) and t is the test duration (hour).

6.3.4 – Migration Cell

The diffusion coefficients for specific ionic species were measured using a migration cell (Stadium Cell) as shown in Figure 20. The test method consists of monitoring the intensity of electrical current passed through a cylindrical test specimen $50 \pm 2 \text{ mm}$ (2 in) thick and $100 \pm 2 \text{ mm}$ (4 in) in diameter over a 14 day testing period. Before testing, the samples are vacuum saturated with 0.3M NaOH for approximately 18 hours. After vacuum saturation, the sample was mounted in between the downstream cell filled with 0.3M NaOH solution and the upstream cell filled with 0.5M NaCl + 0.3M NaOH solution. A constant DC potential of 20 V was maintained across the specimen. The data (voltage, current, and temperature) are automatically recorded at 15 minute intervals. These data, along with the porosity (volume of permeable voids) value determined in accordance with ASTM C642-06, were entered into STADIUM Lab software to evaluate the ion diffusion coefficients (Samson et al., 2003).

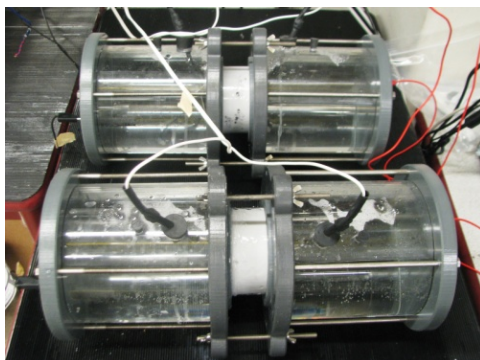


Figure 20: Migration cell experimental set up.

6.3.5 – Chloride Ponding and Profiling

The penetration of chloride ions into concrete was assessed from a chloride ponding test in which a 3% NaCl solution was ponded on the surface of the specimen following the approach described in ASTM C1543-10. A cylindrical specimen was prepared that was 102 mm (4 in) diameter and 203 mm (8 in) long and allowed to cure for 28 days sealed in plastic bags stored at 23 ± 1 °C. After 28 days, the sample was cut obtaining two half cylinders that were 102 mm (4 in) diameter and 102 mm (4 in) long. The sides of each concrete specimen were coated with epoxy. After the epoxy hardened and dried, a plastic cylinder was affixed around the top of each sample, in order to form a dam to contain the salt solutions to be used for ponding. A sketch of the sample is shown in Figure 21.

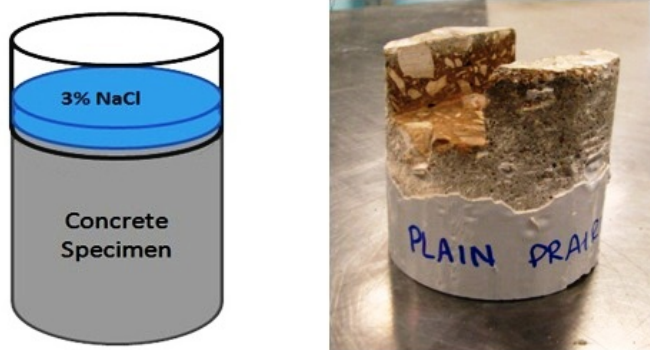


Figure 21: A sketch of the sample used for chloride ponding and the sample after profile grinding.

After the dam was affixed to the surface, water was placed on the specimen to insure that the dam was water-tight. The water was then removed and the sample was filled with a 3% NaCl solution. It should be noted that water could be reabsorbed by the sample, however the samples were not vacuum saturated. The samples were stored in a chamber at 23 ± 1 °C and $50 \pm 2\%$ RH. Periodically (approximately every 10 days), the salt solution was replaced with fresh solution. Once the testing ages were reached (28, 56 and 91 days) the plastic dam was removed and the epoxy sealed sides were removed. The concrete was placed on a milling machine and ground in successive 2 mm increments using a 50 mm diamond tipped drill bit. The powder that was collected at different depths was analyzed to determine the chloride content as described in the following section.

An automated system was used to titrate up to 14 samples (a sample here refers to the powder obtained at each grinding depth) simultaneously (Figure 22). The acid-soluble chloride content was determined using a procedure similar to AASHTO T 260; however some modifications to the specification were adopted and are described in great detail in Di Bella et al. (2012).



Figure 22: Automated titration unit.

7. – Experimental Results

7.1 – Compressive Strength

Compressive strength is generally the most frequently measured property for quality control. Figure 23 shows the compressive strength of the plain and internally cured concrete using the samples that were prepared in the field at the time the decks were cast. During the first week, the internally cured concrete showed a lower compressive strength development (approximately 5%), but after 2 weeks, the strength of the internally cured concrete increased and exceeded the plain concrete. After 3 months, the compressive strength of the internally cured concrete was about 20% higher than the plain concrete because of the continuous hydration of the mixture at later ages, promoted by the extra water stored in the LWA.

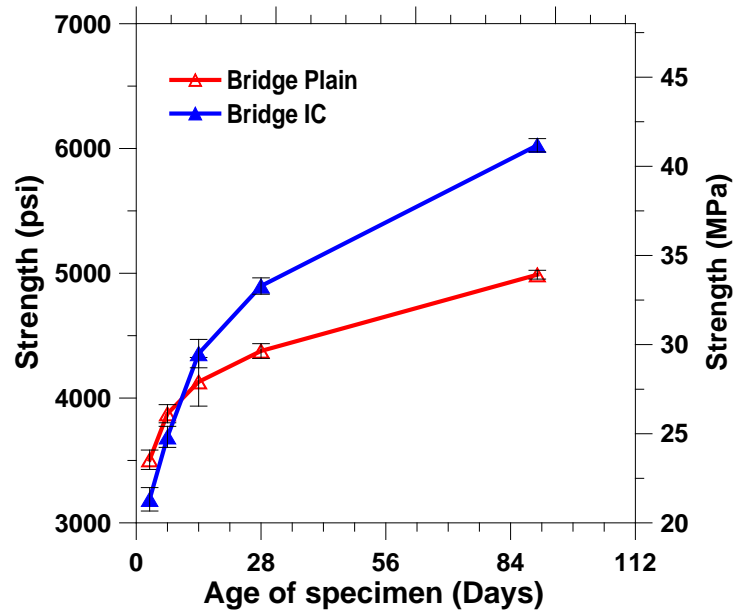


Figure 23: Compressive strength of the plain vs. internally cured concrete as cast on site.

Since concrete is a composite material, its compressive strength, as well as most of its properties, are influenced by each individual component. Even though lightweight aggregate can be considered the weakest component in concrete the compressive strength of the internally cured concrete is higher than the plain concrete (Wassermann et al., 1996). In addition to an increased hydration and hence a denser microstructure, an increased compressive strength in internally cured concrete has been associated with a denser ITZ, and less early age cracking (Lura, 2003; Golias, 2010).

7.2 – Volume Change

Concrete undergoes volume changes throughout its life that can take place under different conditions (i.e. sealed and unsealed). Volume changes themselves are not a problem for the concrete; however, if these volume changes are restrained, by the box girders for example, stresses can develop in the concrete leading to cracking. Many elements act as constraints, such as subgrades (for a pavement slab), reinforcing steel, as well as new concrete poured adjacent to an old concrete element (Schlitter, 2010). In this work, the main restraint comes from the precast box girders.

7.2.1 – Unrestrained Linear Autogenous Shrinkage—Large Tube

Based on the same protocol developed for cement paste (Jensen et al., 1995), autogenous strain of concrete was monitored using a special mold. The results of the unrestrained shrinkage tests performed on the plain and internally cured concrete mixtures are shown in Figure 24. The measured strains were zeroed at setting (corresponding to approximately 4 h).

The plain mixture shows a slight expansion right after setting while shrinkage at later ages takes place. The internally cured mixture, instead, shows a reduced autogenous strain development.

It should be noted that the difference in strain between the plain and internally cured mixture is unexpectedly high. This can be addressed to the fact that many artifacts can be encountered when autogenous shrinkage is measured directly in concrete.

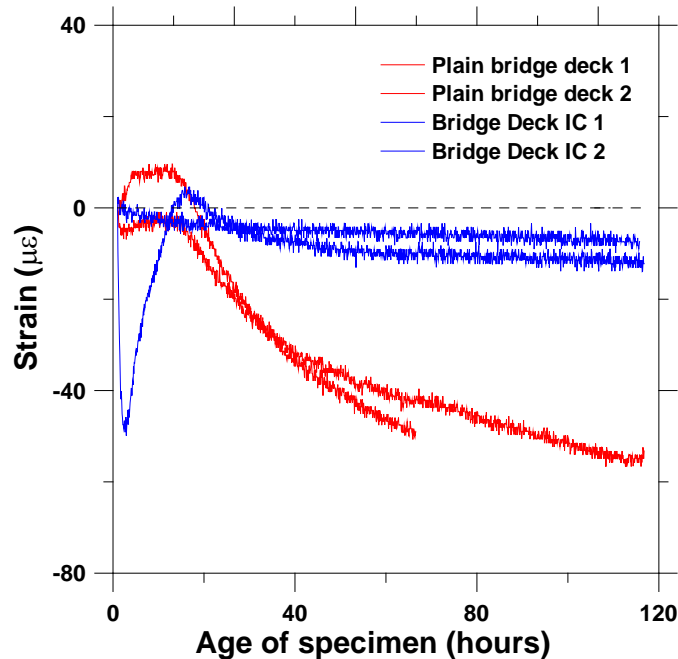


Figure 24: Autogenous shrinkage results of plain and internally cured concrete mixtures.

7.2.2 – Restrained Shrinkage – Dual Ring Test

The restrained dual ring test was utilized to study the restrained shrinkage behavior of the plain and internally cured bridge deck mixtures. Two representative mortar mixture designs were proportioned for the dual ring that utilized the same paste content and utilized the constituent materials collected at the ready-mix plant. The residual stress histories are shown in Figure 25. During the first 4-days, the temperature in the chamber was kept constant at 23 ± 1 °C. Therefore, the development of residual stress was due to autogenous shrinkage only. It can be seen that during the period of constant temperature, the internally cured mixture developed a slight expansion at a very early age, as evidenced by negative residual stress. The mixture subsequently shrank. This early age stress history agrees with the initial expansion and subsequent shrinking that was measured in the unrestrained linear autogenous shrinkage test. At the end of the period of constant temperature, the internally cured mixture developed 39% less residual tensile stress as compared to the plain mixture. This reduction can be attributed to the reduced amount of shrinkage exhibited by the internally cured mixture as a result of the prewetted lightweight aggregates mitigating the effects of self-desiccation. Further, the internally cured mixtures would also be expected to exhibit greater stress relaxation due to a lower modulus of elasticity from the inclusion of the softer LWA (Schlitter et al., 2010; Raoufi et al., 2011; Barrett et al., 2011).

The sharp increase seen in the plot of residual stress after four days (96 hours) was due to the temperature reduction that was imposed on the system which generated thermal shrinkage of the specimen. It should be noted that the restraining rings are constructed of a thermally stable material, Invar, so that the restraint is stable throughout a temperature change. This method of imposing thermal stress with a temperature reduction can be used to quantify the reserve stress of the specimen, or in other words, how close the specimen is to cracking.

The internally cured concrete is shown to develop less thermal stresses compared to the plain concrete.

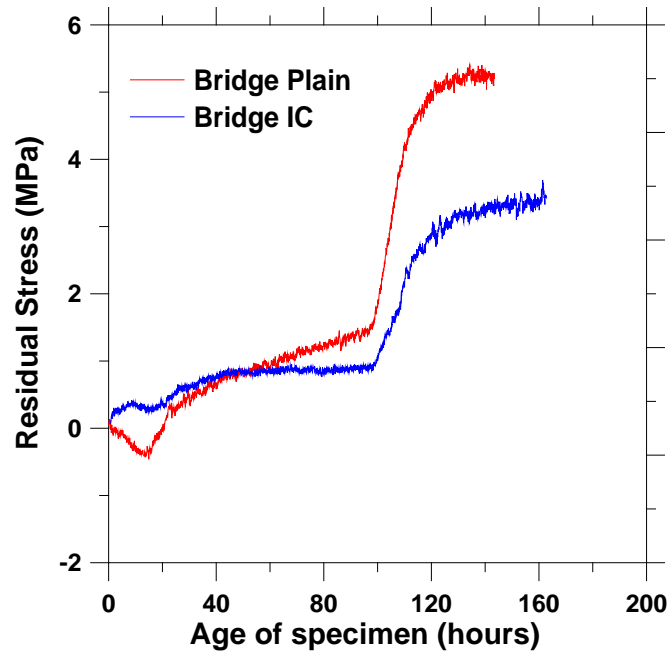


Figure 25

None of the samples cracked at the minimum temperature reached -5°C . However, from the two plots, it can be concluded that the plain concrete would have a greater potential to crack than the internally cured concrete.

7.2.3 – Drying and Autogenous Shrinkage

While autogenous shrinkage is defined for a sealed system, drying shrinkage is a volume change due to a loss of water (unsealed system). Drying is very often erroneously considered to be a change of volume only due to evaporation, where it is a combination of self-desiccation (internal drying) and evaporation (external drying) (Randliska et al., 2008).

The unrestrained length change results are shown in Figure 26 as average of three different samples for each mixture. The internally cured concrete shrinks at a slower rate and shows an overall lower shrinkage (approximately 25% at 28 days and 17% at 91 days). A reduction in volume change is beneficial since it will reduce the likelihood for cracking.

It should be noted that even though the internally cured concrete shrinks less, more water through evaporation is lost (Figure 27), which is consistent with the findings of Henkensifken et al. (2009).

Figure 27 presents the mass loss due to evaporation (drying exposure). As expected, the lightweight aggregate mixture shows an overall reduced shrinkage despite a higher weight loss. The system contains extra water that is stored in the largest pores of the LWA. Since the LWA pores are larger than those in the hydrating cement paste, water is preferentially drawn from the LWA to the surrounding paste and the emptying of these larger pores produces a much lower capillary stress, which reduces the strain and therefore the likelihood for early age cracking (Bentz, 2010).

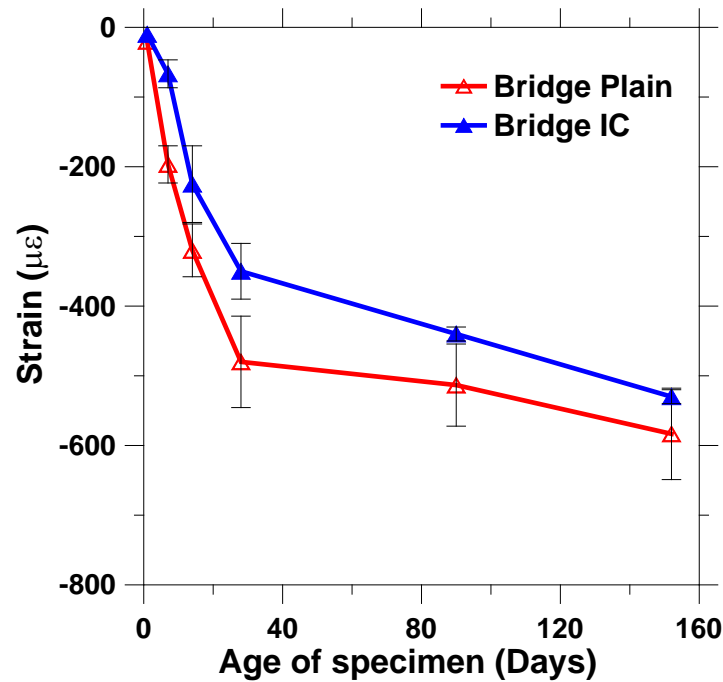


Figure 26: Free shrinkage in unsealed curing conditions.

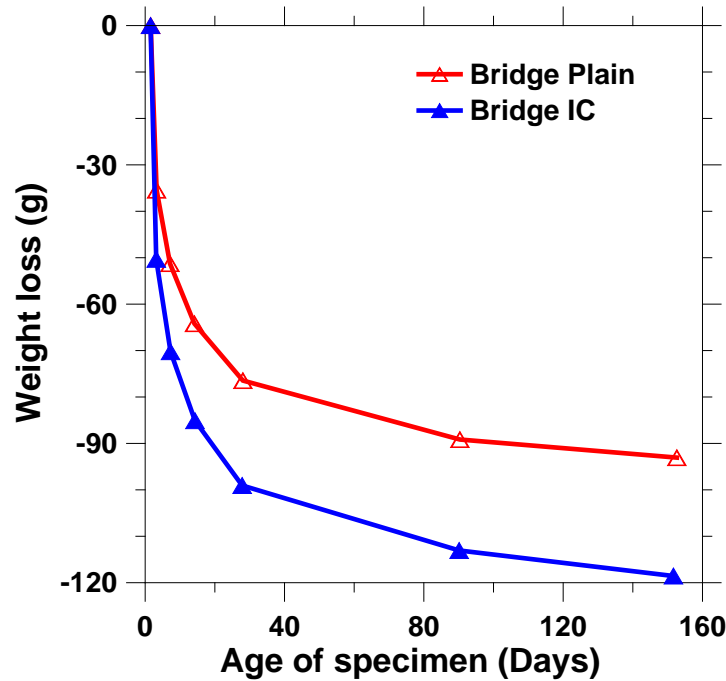


Figure 27: Mass loss of free shrinkage samples in unsealed curing conditions.

7.3 – Transport Properties

7.3.1 – Rapid Chloride Penetration Test (RCPT)

The most widely used method by transportation agencies to assess concrete's ability to resist chloride ion penetration is described in ASTM C 1202-12 or AASHTO T 277.

The penetrability of concrete itself depends only on the pore volume and pore network of the concrete. In the RCP test, however, the current that passes through the sample during the test indicates the movement of all the ions in the pore solution that essentially represents the sample conductivity (Snyder et al., 2000; Joshi et al., 2002). The conductivity depends on the pore structure and the chemistry of the pore solution (Shi, 2004).

The rapid chloride penetrability test was carried out on laboratory prepared samples cured in sealed bags for until time of testing. The samples were then vacuum saturated. The results from the rapid chloride penetrability and the temperature at the end of the 6 hours testing period are shown in table 4 for the plain and internally cured concrete. The internally cured mixture shows consistently lower charge passed at any age. For example at an age of 91 days the internally cured concrete has an RCP value that is approximately 35% lower than the plain concrete and that becomes 55% lower than the plain concrete at the age of 180 days.

Table 4: Charge Passed at Six Hour Test and Corresponding Temperature for the Plain and Internally Cured Concrete over Ten Months

	Monroe County Bridge Deck Concrete			
	Charge Passed [Coulombs]		Final Temperature [°C]	
Time [day]	Plain	IC	Plain	IC
28	4252	3822	44	45
56	2863	2458	33	35
91	3174	2065	33	35
180	2656	1239	32	28
300	*	1080	*	28

* No values are available

The use of the rapid chloride penetration test method can be thought of essentially as a measure of concrete resistivity. However, it should be noted that RCPT was performed with high voltage and the sample heated during testing which increased due to the Joule effect. As such there is no need to continue the test for six hours, and changes in the current during this time are most likely due to increases in temperature, not chloride penetration (Betancourt et al., 2004; Snyder et al., 2000).

Despite its unreliability, the RCP test is still considered by many DOT's to be a valid test for determining the ability of concrete to resist chloride ingress and it is still a part of quality control (Pfeifer et al., 1994).

7.3.2 – Surface Resistivity

The electrical resistance of the concrete is known to be related to the pore volume, the ionic concentration of pore solution (Kessler et al., 2005), the degree of saturation in the concrete, and the tortuosity of the pore network (Spragg et al., 2011). The resistivity of the concrete can be used as a surrogate measurement of the transport properties since the Nernst-Einstein relationship (Garboczi, 1990) allows the electrical properties to be related to ionic diffusion.

Figure 28 shows the surface resistivity measurements for plain and internally cured samples. Initially, the internally cured concrete has a lower resistivity (likely due to the conductive nature of the aggregate [Di Bella et al., 2012]); however by 56 days the resistivity of the internally cured concrete has a similar resistivity to that of the plain concrete, and at later ages it is more resistive thanks to the enhanced hydration and denser ITZ (Bentz, 2009; Castro, 2011).

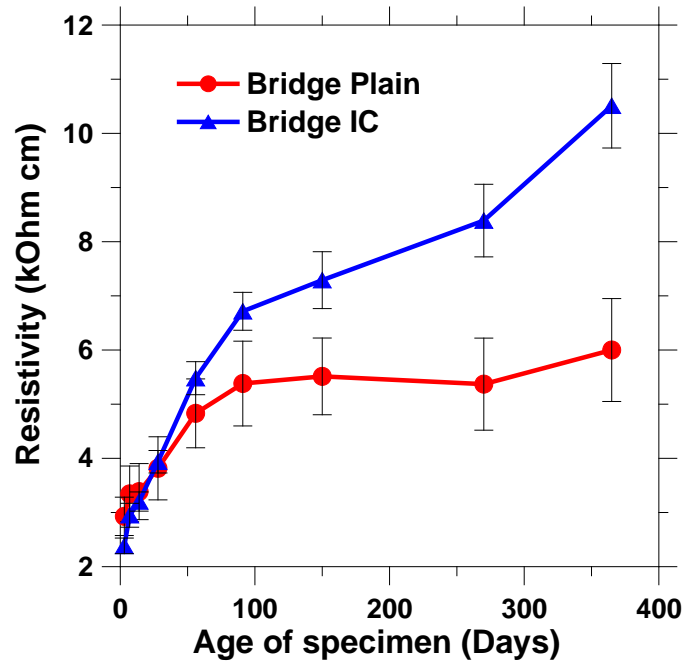


Figure 28: resistivity measurement with a Wenner array.

It should be noted that by storing the samples in lime water they will hydrate more than a sealed sample. While the surface resistivity test is easy to perform, the samples are water saturated through the test which may not represent the way these specimens are cured in the field (Golias, 2010).

7.3.3 – Rapid Migration Test

Diffusion coefficients are one of the most important parameters that need to be considered in predicting service life. Different methods are described in literature to determine the diffusion coefficient (Tang, 1997).

Table 5 shows the non steady migration coefficients obtained at 28, 56, 91, 180 and 300 days for the concrete samples with and without internal curing. The internally cured concrete shows benefits of internal curing for each test compared to the plain concrete. For example, at an age of 91 days the internally cured concrete has an RCP value that is approximately 15% lower than the plain concrete. This difference increases at 180 days in which the internally cured concrete has chloride diffusion coefficient that is up to 30% lower than the plain concrete.

Table 5: Chloride Diffusion Coefficients Obtained from the Rapid Chloride Migration Test

	Monroe County Bridge Deck Concrete	
Time [day]	Diffusion coefficients (m²/s)	
	Plain Concrete	IC Concrete
28	1.42E-11	1.15E-11
56	1.26E-11	8.98E-12
91	3.99E-12	3.42E-12
180	4.7E-12	3.32E-12
300	*	3.2E-12

* No values are available

Although the internally cured concrete shows lower diffusion coefficients than the plain concrete from the test when it is performed following the standard, it should be noted that the diffusion coefficients may actually be lower for the internally cured concrete. First, the internally cured concrete is cut to perform the test. As such, the cutting of the concrete exposes the porous lightweight aggregate. When these cut aggregates are exposed to the solution, the chloride can easily diffuse into the concrete which may not represent what happens in field concrete. As such this may skew the results of test with more resistant matrices (Bentz et al., 2011; Di Bella et al., 2012). In addition, since the sample is saturated, the conductivity of the aggregate may alter the electrical response of the concrete (Weiss et al., 2012). Despite these testing anomalies the internally cured concrete performed as well, or better than the plain concrete.

7.3.4 – Migration Cell

A multi-ionic model considers the electrical coupling between ions, chloride binding, and chemical reactions which were used to interpret the results from the migration cell (Samson et al., 2003). This analysis was performed using a program called STADIUM lab which used results from the migration cell along with the porosity results obtained from ASTM C 642-06 (Table 6). The modeled diffusion coefficients in Table 7 have a similar trend when compared with the rapid chloride penetration test; however the diffusion coefficients obtained with this method are not directly comparable with one another as shown.

The differences in the diffusion coefficients between the plain and internally cured concrete obtained by Stadium IDC are greater than that observed from the NT Built test method. For example, at an age of 91 days the internally cured concrete has an RCP value that is approximately half that of the plain concrete.

Table 6: Tortuosity Modeled by Stadium Lab Software and Porosity Determined in Accordance with ASTM 642

	Monroe County Bridge Deck Concrete			
Time [days]	Porosity %		Tortuosity	
	Plain Concrete	IC Concrete	Plain Concrete	IC Concrete
28	12.6	13.0	0.0421	0.0284
91	13.3	14.5	0.0377	0.0146

Table 7: Chloride Diffusion Coefficients Obtained Using the Migration Cell

	Monroe County Bridge Deck Concrete	
Time [day]	Diffusion coefficients (m²/s)	
	Plain Concrete	IC Concrete
28	8.56E-11	5.78E-11
91	7.67E-11	2.97E-11

The porosity and the pore characteristics of the concrete play a fundamental role in the permeability and in general on the durability of the concrete. The effects of porosity and pore characteristics can be captured through a single parameter called tortuosity (Mason, 1997; Ahmad et al., 2005). STADIUM Lab also provides a measure of the tortuosity of the pore structure in concrete as shown in Table 6. While the porosity for internally cured concrete is higher due to the presence of the lightweight aggregates, the tortuosity of the internally cured concrete is lower than that of the plain concrete presumably due to the increased hydration and densified interfacial transition zone in internally cured concrete.

7.3.5 Chloride Ponding and Profiling

Figures 29 through 31 show the acid-soluble chloride content of samples ponded with a 3% NaCl for a period test of 28 and 91 days, respectively.

Figure 29 shows the chloride content for the plain concrete and the internally cured concrete ponded for 28 days. Within the first 8-10 mm, the chloride concentration is greater in the internally cured mixture. This could be explained by the fact that the samples are cut, thereby exposing the pores of the lightweight aggregates at the surface of the sample. The chloride solution can be absorbed or diffuse into the LWA pores rapidly.

To confirm the artifact of the test method, two samples were prepared, a plain concrete and an internally cured concrete at an age of 91 days. The samples were not exposed to water as the other samples were; however, they were ponded for 15 minutes with a sodium chloride solution. The solution was then removed and the samples were ground. Figure 30 shows that both concretes absorb fluid. The internally cured concrete however absorbs more solution near the surface due to pores of the LWA being connected to the surface. Field concrete however does not have exposed aggregate and it appears that the increased concentration of the chloride at the surface is in part an experimental artifact of the test method. Additional testing is being done to

better quantify this effect. Consequently, due to the higher concentration at the surface in the internally cured concrete it could be expected that the chloride concentration would have been for the entire profile higher compared to the plain concrete. However, as it can be seen from Figure 29 at depths greater than 8-10 mm the concentration of chloride is similar for both mixtures. This confirms the results above where the diffusion coefficient for the internally cured concrete was found to be lower.

Figure 31 shows the experimental data after 91 days of ponding. The results show a similar trend observed for the mixtures ponded for 28 days with the internally cured concrete again showing a higher chloride concentration within the first 8 mm.

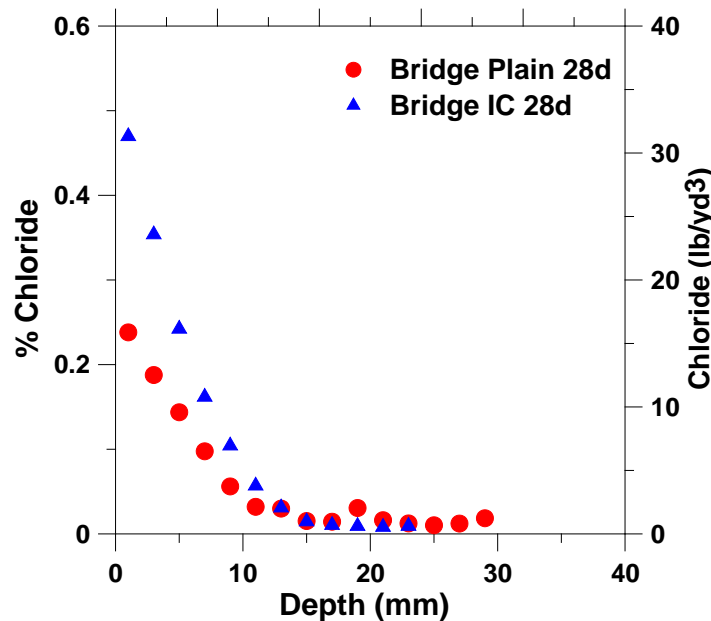


Figure 29: Chloride content in a plain and internally cured concrete after 28 days of ponding.

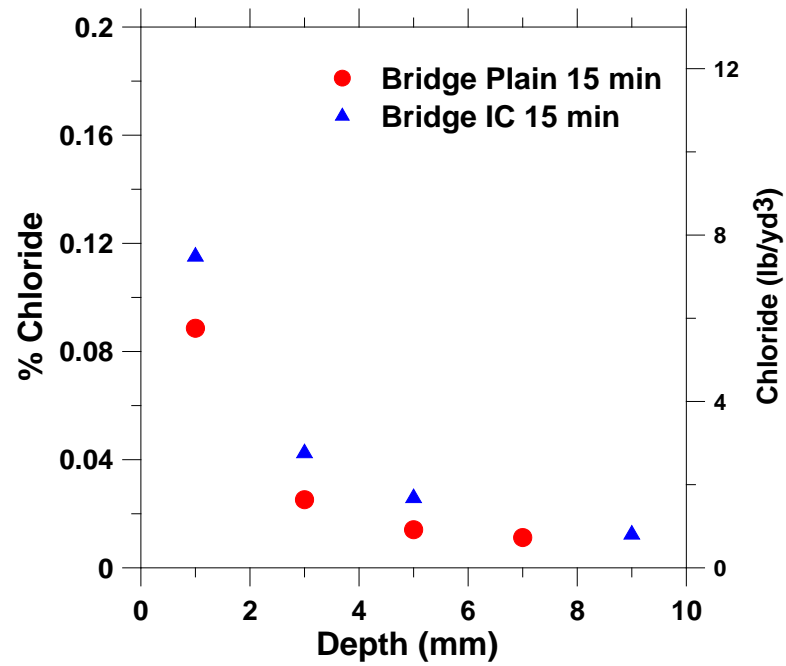


Figure 30: Chloride content in a plain and internally cured concrete at 91 days after only 15 minutes of ponding.

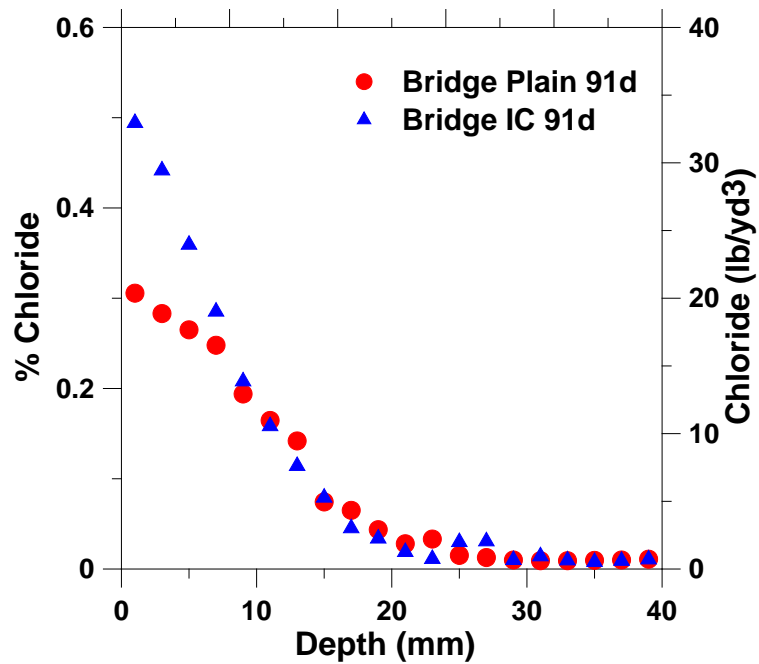


Figure 31: Chloride content in a plain and internally cured concrete after 91 days of ponding.

8. – Visual Inspection of the Bridge after 12 and 20 Months from the Casting

One year after casting the two bridge decks, a visual inspection was performed. The plain bridge deck contained two long cracks: one longitudinal and the other one transverse. On the other hand the internally cured deck did not have any visible cracking.

The crack that developed in the plain concrete bridge deck developed in the region between two box girders. In Figure 32 and 33, the longitudinal and transverse cracks for the plain bridge deck are shown respectively. The length of the two cracks is noticeable. In fact, both of them reach the middle of the bridge deck.

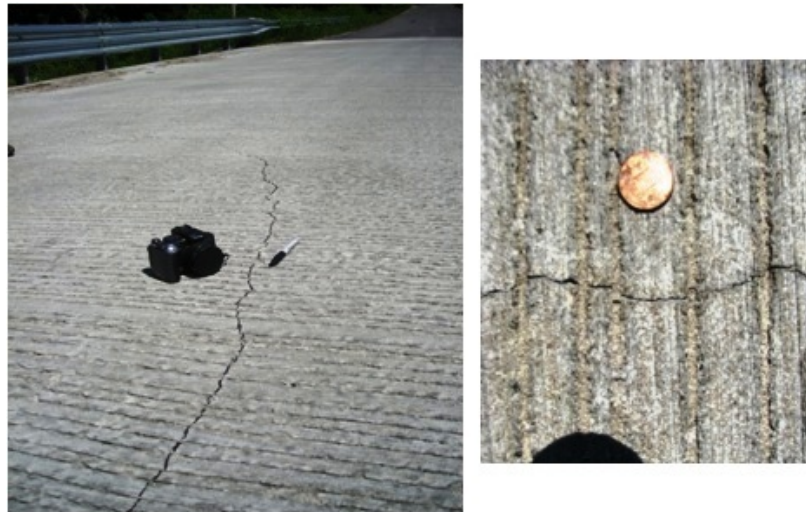


Figure 32: Longitudinal crack developed in the plain bridge deck.

In Figures 33 and 34 are shown the position and length of the transverse crack developed on the plain bridge deck. Figure 34, in addition, shows that the crack went through the overlay and reached the top of the box girders.



Figure 33: Transverse crack developed in the plain bridge deck



Figure 34: Plain Bridge deck and enlargement of a portion showing the transverse crack going through the deck.

Since the bridge decks are close to each other it is assumed that the volume of traffic and environmental conditions are similar. When a visual inspection of the bridges was conducted (September 2011 and May 2012) it was concluded that the internally cured bridge deck was performing better than the conventional bridge deck. This was consistent with the experimental results from laboratory testing.

The superior performance of the internally cure bridge deck mixture resulting in no visible cracks will greatly positively affect the durability and service life of the bridge deck. Even very small cracks contribute to speed up the ingress of aggressive species such as chlorides as well as dust and water that are able to compromise the durability of the bridge deck.

Finally, in Figure 35 is shown the internally cured bridge deck which didn't reported any visible cracking at the time of the visual inspection.



Figure 35: Internally cured bridge deck; which did not show cracking.

9. – Conclusions

This report presents documentation about the construction of two different concrete bridge decks in Monroe County, Indiana. The bridges were cast in close proximity to one another by the same contractor using similar materials and construction methods on consecutive days to obtain a direct comparison of the two mixtures. The first bridge deck was made using conventional concrete (i.e., plain concrete) that satisfied typical INDOT specifications while the second bridge deck was made using a similar mixture however the concrete had a portion of the fine aggregate replaced with an equivalent volume of prewetted lightweight aggregate to make an internally cured concrete.

The unique feature of this study consists in the evaluation and comparison of the two bridge decks that were cast close to each other and subjected to the same environment and traffic allowing an investigation of the impact of internal curing in the field practice.

Samples from these mixtures were used to evaluate a variety of concrete performances. Tests were focused both on early age properties as well as long term properties: compressive strength, volume change properties: 1) autogenous and 2) drying shrinkage and chloride transport performance using a series of tests including: 1) resistivity, 2) rapid chloride penetration (RCP), 3) rapid chloride migration (the Nord Test), 4) migration cell testing and 5) chloride ponding and profiling. Tests were performed 28, 56, and 91 days after casting.

All of the test results show in a very clear fashion that the use of lightweight aggregate in concrete as internal curing agent is beneficial for the all the aspects investigated in this study.

At 90 days the compressive strength for the internally cured concrete resulted in values 20% higher when compared to the plain concrete thanks to an improved cement hydration. This improved hydration can also reduce microcracking as a result of the lower shrinkage tendency of concrete with lightweight aggregate used for internal curing.

A reduction in volume change was observed in the case of the internally cured concrete. A reduction in autogenous shrinkage development was also observed in the case of the internally cured mixture. In addition, these results were confirmed by the fact that the internally cured mixture showed a reduced residual stress development compared to the plain mixture. Further, the internal curing reduced the residual stresses developed by thermal changes. Drying shrinkage for the internally cured mixture resulted in values lower than those of the plain concrete by approximately 27% after 28 days and 11% after 150 days.

The diffusion coefficients measured included the migration cell (STADIUM cell) and the rapid chloride migration test showed that the internally cured concrete had a lower diffusion coefficient than the plain concrete (15% and 50%, respectively). The rapid chloride penetrability of the internally cured concrete is lower than the plain concrete at all of the ages tested (approximately 55% at 180 days). The electrical surface resistivity of the internally cured concrete is higher than the plain concrete (approximately 45% higher at 365 days). These results all show a similar trend. As such, the internally cured concrete has the ability to reduce transport properties. It is believed that this is due to increased cement hydration and reduced porosity at the interfacial zone based on earlier studies.

While the tests showed benefits of using internal curing, it should be noted that some artifacts are believed to exist in the current tests that are caused by the presence of the prewetted lightweight aggregates. The use of the cut surface in samples prepared for the rapid chloride penetration (RCP), rapid chloride migration, migration cell testing and chloride ponding and profiling enable chloride to enter the lightweight aggregate at the surface which appears to influence the testing results. This result much more evident in the chloride ponding where higher chloride concentration on the top of the sample was observed. For example, the chloride penetration was observed to be higher in the internally cured concrete in the 6 to 8 mm near the surface. The diffusion coefficient is however lower in the IC mixtures. In addition, tests that use vacuum saturation enable water to fill the lightweight aggregate which allows them to behave as electrical conductors which reduces the resistivity of the concrete. Procedures are currently being developed to quantify these effects and to develop methodologies to account for these artifacts.

A visual inspection, performed a year after the casting of the two bridge decks, brought to light the presence of two long cracks in the plain bridge deck. On the other hand, the internally cured bridge deck did not have any visible cracking.

The use of lightweight aggregate as an internal curing agent is recommended for bridge deck concrete applications. It can be particularly beneficial in high performance concrete applications to offset their natural inclination for autogenous shrinkage. As a result of the information identified as a part of this research, bridge deck concretes that have typical water contents can also benefit from the use of lightweight aggregate in an internal curing application. It must also be noted that in addition to the internal curing, effective external curing is necessary to increase the performance of the bridge deck concretes, ultimately resulting in an increased service life.

10. – Appendix

SPECIAL PROVISIONS

SP40. PURDUE UNIVERSITY RESEARCH, BRIDGE 61

Description: The Contractor shall provide access to Purdue University before and during pouring of the Monroe County Bridge 61 bridge deck. This work requires coordination and cooperation between the Contractor and representatives from Purdue University.

Materials: The research instrumentation will be provided by Purdue University.

General Requirements: Purdue University will be performing instrumentation and material testing of the bridge deck concrete on Monroe County Bridge 61. The locations of the testing will be as determined by Purdue University. This work involves construction activities at the job site during and after the deck construction.

1. The Contractor will place the I-button instruments provided by Purdue University at the locations designated by Purdue. Alternatively, Purdue University will install instrumentation provided the Contractor provides fall safety protection access as necessary.
2. The Contractor shall notify Purdue University if any research instrumentation is disturbed during construction.
3. The Contractor shall make provisions to continue the wires for the instrumentation through the deck formwork to provide Purdue University access to the wires after the deck pour for future measurement.
4. The Contractor shall take all necessary precautions to prevent damage to the research instrumentation and cooperate with the representatives from Purdue University during the installation and monitoring of the research instrumentation and during their testing in accordance with 105.07.
5. The Contractor shall provide the Engineer five business days notice prior to the placement of the deck reinforcement and five business days notice prior to the casting of the concrete deck for Monroe County Bridge 61. The Engineer will be responsible to contact Purdue University.

Jobsite Activities and Requirements: If the Contractor elects to convey concrete by the means of pumping for the placement of Special Concrete, C, Superstructure, Bridge 61, the Contractor shall notify the Engineer and shall follow Special Provision 42. The Engineer shall notify Purdue University that the Contractor is utilizing concrete pumping.

Pumping will not be permitted at the research instrumentation locations.

SPECIAL PROVISIONS

Purdue University will perform independent concrete testing during the placement of the deck concrete. Purdue University will cast concrete cylinders (approximately 60) and perform other tests (e.g. 2 cylinders for RCPT and six prisms for shrinkage) on the concrete used in the bridge deck. Less than a cubic yard of concrete would be needed for Purdue University testing. Purdue University testing does not replace any of the other testing required by the Contractor. The Contractor shall perform all testing and sampling of materials as specified in Special Provision No. 17.

Purdue University shall be notified of the date and location of the preconstruction conference.

Placement of the deck concrete shall not begin until Purdue University has deemed instrumentation of the bridge deck complete.

The contact for Purdue University is:

Jason Weiss, Ph.D.
 Professor of Civil Engineering
 Director of Pankow Materials Laboratory
 Purdue University School of Civil Engineering
 550 Stadium Mall Drive
 West Lafayette, IN 47907
 (765)-494-2215
 (765)-494-0395 (Fax)
 Email: wjweiss@purdue.edu

Method of Measurement: This work will not be measured.

Basis of Payment: This work will not be paid for directly, but shall be included in the cost of the Special Concrete, C, Superstructure, Bridge 61.

SP41. PURDUE UNIVERSITY RESEARCH, BRIDGE 49

Description: The Contractor shall provide access to Purdue University before and during pouring of the Monroe County Bridge 49 bridge deck. This work requires coordination and cooperation between the Contractor and representatives from Purdue University.

Materials: The research instrumentation will be provided by Purdue University.

General Requirements: Purdue University will be performing instrumentation and material testing of the bridge deck concrete on Monroe County Bridge 49. The locations of the testing will be as determined by Purdue University. This work involves construction activities at the job site during and after the deck construction.

SPECIAL PROVISIONS

1. The Contractor will place the I-button instruments provided by Purdue University at the locations designated by Purdue. Alternatively, Purdue University will install instrumentation provided the Contractor provides fall safety protection access as necessary.
2. The Contractor shall notify Purdue University if any research instrumentation is disturbed during construction.
3. The Contractor shall make provisions to continue the wires for the instrumentation through the deck formwork to provide Purdue University access to the wires after the deck pour for future measurement.
4. The Contractor shall take all necessary precautions to prevent damage to the research instrumentation and cooperate with the representatives from Purdue University during the installation and monitoring of the research instrumentation and during their testing in accordance with 105.07.
5. The Contractor shall provide the Engineer five business days notice prior to the placement of the deck reinforcement and five business days notice prior to the casting of the concrete deck for Monroe County Bridge 49. The Engineer will be responsible to contact Purdue University.

Jobsite Activities and Requirements: If the Contractor elects to convey concrete by the means of pumping for the placement of Special Concrete, C, Superstructure, Bridge 49, the Contractor shall notify the Engineer and shall follow Special Provision 43. The Engineer shall notify Purdue University that the Contractor is utilizing concrete pumping.

Pumping will not be permitted at the research instrumentation locations.

Purdue University will perform independent concrete testing during the placement of the deck concrete. Purdue University will cast concrete cylinders (approximately 60) and perform other tests (e.g. 2 cylinders for RCPT and six prisms for shrinkage) on the concrete used in the bridge deck. Less than a cubic yard of concrete would be needed for Purdue University testing. Purdue University testing does not replace any of the other testing required by the Contractor. The Contractor shall perform all testing and sampling of materials as specified in Special Provision No. 17.

Purdue University shall be notified of the date and location of the preconstruction conference.

Placement of the deck concrete shall not begin until Purdue University has deemed instrumentation of the bridge deck complete.

See Special Provision 40 for the contact information for Purdue University.

SPECIAL PROVISIONS

Method of Measurement: This work will not be measured.

Basis of Payment: This work will not be paid for directly, but shall be included in the cost of the Special Concrete, C, Superstructure, Bridge 49.

SP. 42 SPECIAL CONCRETE , C, SUPERSTRUCTURE, BRIDGE 61

Description: This work shall consist of furnishing and placing Special Concrete, C, Superstructure, Bridge 61 in the concrete deck for Monroe County Bridge 61 as shown in the plans in accordance with these Special Provisions and Section 702 of the Standard Specifications. The Special Concrete, C, Superstructure, Bridge 61 will have an adjusted water to cement ratio and 380 lb/yd³ of the fine aggregate replaced with an equivalent volume of an approved water carrying fine lightweight aggregate (with between 11 and 15% absorption at 24 hrs). Recommendations on approved fine lightweight aggregates can be obtained from Purdue University.

Material: The material in these items shall be in accordance with Section 702 of the Standard and Supplemental Specifications except as modified below.

General Requirements: The following Special Concrete, C, Superstructure, Bridge 61 Specification shall be used for the concrete deck for Monroe Co. Bridge 61 within this project.

SECTION 702, LINE 20, INSERT AS FOLLOWS:

pound (kilogram) of cement	0.490	0.620	0.443 0.403
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The target water cement ratio shall be 0.383

SECTION 702, LINE 66, INSERT AS FOLLOWS:

The producer shall use 380 lb/yd³ of fine lightweight aggregate as a replacement for an equivalent volume of their normal weight fine aggregate. This weight of lightweight aggregate can be reduced for aggregates with 24 hour water absorption greater than 11%. The lightweight aggregate used shall meet ASTM C33 requirements and shall be approved by the Engineer. The Engineer shall receive recommendations from Purdue University for approved lightweight aggregate.

SECTION 702, LINE 95, INSERT AS FOLLOWS:

This requirement shall be based only on the normal weight reference mixture.

SECTION 702, LINE 117, INSERT AS FOLLOWS:

The Contractor shall submit the Special Concrete, C, Superstructure, Bridge 61 mix design to the Engineer for approval. The Engineer will submit the mix design to the representative from Purdue University for review.

SPECIAL PROVISIONS

SECTION 702, LINE 205, INSERT AS FOLLOWS:

The fine lightweight aggregate shall be batched in a 24 hour saturated condition.

SECTION 702, LINE 475, INSERT AS FOLLOWS:

of the air content and the water content. Modifications to the water carrying fine lightweight aggregate used may be required if the concrete is to be pumped. Purdue University shall provide the additional water carrying fine lightweight aggregate replacement amounts using the approved water carrying fine lightweight aggregate specified.

SECTION 702, LINE 1168, INSERT AS FOLLOWS:

Evaporation retardant shall be applied in accordance with the manufacturer's recommendations immediately after screeding and re-applied when the surface is disturbed, such as during tining, prior to final cure. The evaporation retardant shall be the product of one of the following manufacturers or approved equal. Certification shall be in accordance with Article 912.03(c)2.

1. MASTER BUILDERS TECHNOLOGIES. The evaporation retardant shall be Confilm as manufactured by Master Builders Technologies, 3715 Bargetown Road, Room 214, Louisville, Kentucky 40218.
2. SIKA CORPORATION. The evaporation retardant shall be Sika-Film as manufactured by Sika Corporation, 2930 Switzer Road, Columbus, Ohio 43219.
3. EUCLID CHEMICAL COMPANY. The evaporation retardant shall be Eucobar as manufactured by Euclid Chemical Company, 19218 Redwood Road, Cleveland, Ohio 44110.

SECTION 702, LINE 1168, INSERT AS FOLLOWS:

If cracks are found, the contact at Purdue University detailed in Special Provision 40 shall be notified.

Method of Measurement: Special Concrete, C, Superstructure, Bridge 61 will be measured by the cubic yard.

Basis of Payment: The accepted quantities of Special Concrete, C, Superstructure, Bridge 61, all work, materials and equipment necessary as described herein are to be included in the cost of "Special Concrete, C, Superstructure, Bridge 61" as provided in the Itemized Proposal.

SPECIAL PROVISIONS

SP. 43 SPECIAL CONCRETE , C, SUPERSTRUCTURE, BRIDGE 49

Description: This work shall consist of furnishing and placing Special Concrete, C, Superstructure, Bridge 49 in the concrete deck for Monroe County Bridge 49 as shown in the plans in accordance with these Special Provisions and Section 702 of the Standard Specifications. The Special Concrete, C, Superstructure, Bridge 49 will have an adjusted water to cement ratio.

Material: The material in these items shall be in accordance with Section 702 of the Standard and Supplemental Specifications except as modified below.

General Requirements: The following Special Concrete, C, Superstructure, Bridge 49 Specification shall be used for the concrete deck for Monroe Co. Bridge 49 within this project.

SECTION 702, LINE 20, INSERT AS FOLLOWS:

pound (kilogram) of cement	0.490	0.620	0.443	0.403
----------------------------	-------	-------	-------	-------

The target water cement ratio shall be 0.383

SECTION 702, LINE 117, INSERT AS FOLLOWS:

The Contractor shall submit the Special Concrete, C, Superstructure, Bridge 49 mix design to the Engineer for approval. The Engineer will submit the mix design to the representative from Purdue University for review.

SECTION 702, LINE 475, INSERT AS FOLLOWS:

of the air content and the water content.

SECTION 702, LINE 1168, INSERT AS FOLLOWS:

If cracks are found, the contact at Purdue University detailed in Special Provision 40 shall be notified.

Method of Measurement: Special Concrete, C, Superstructure, Bridge 49 will be measured by the cubic yard.

Basis of Payment: The accepted quantities of Special Concrete, C, Superstructure, Bridge 49, all work, materials and equipment necessary as described herein are to be included in the cost of "Special Concrete, C, Superstructure, Bridge 49" as provided in the Itemized Proposal.

11. – Acknowledgments

The authors gratefully acknowledge the financial support from the Indiana Local Technical Assistance Program (IN LTAP) and the Expanded Shale, Clay, and Slate Institute (ESCSI).

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